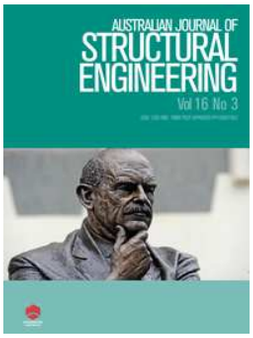

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Shear strength of reinforced sugarcane bagasse ash-laterised concrete beams

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Sugarcane bagasse ash; laterised concrete; shear strength; deflection; beams; shear reinforcement; failure mode

ABSTRACT
The need to utilise waste and locally available materials in the construction industry have necessitated research in them in different parts of the world. This study describes tests on experimental shear behaviour of concrete containing sugarcane bagasse ash and laterite as partial replacement of cement and sand, respectively. Concrete mix of 1:2:4 with a constant slump of 30 ± 2 mm was used to produce 7 beam specimens. The shear span/effective depth ratio was kept constant at 1.5. The results show that (a) the mode of failure of the beams is affected by the partial replacement levels (b) the ultimate cracking load decreases with load deceases with increase in the partial replacement levels (c) the ultimate shearing stress decreases with increase in the partial replacement level (d) all the beams with up to 15% replacement of cement by sugarcane bagasse ash and 20% replacement of sand by laterite soil gave higher shear strength than the control beam without shear reinforcement and finally (e) the presence of the sugarcane bagasse ash and laterite soil improves the deflection at ultimate applied load in comparison with that of the control specimen.

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Introduction

The need for a cheaper, affordable and eco-friendly material in the construction industry has motivated worldwide research regarding blending and partial replacement of the constituent materials of concrete. Laterite has been used in different parts of the world to produce concrete (Lasisi, Osunade, and Adewale 1990; Salau and Sharu 2004; Ata, Olusola, and Aina 2005; Ata and Adesanya 2007; Muthusamy and Kamaruzaman 2012; Ettu et al. 2013; Kamaruzaman and Muthasamy 2013; Ukpata and Ephraim 2013; Jayaraman, Jayaraman, and Saravanan 2014; Muthusamy, Vesuvapateran, and Kamaruzaman 2014). Sugarcane bagasse ash on the other hand has also been used as a pozzolanic material to blend cement in concrete and mortars (Balogun and Adepegba 1982; Ganesan, Rajagopal, and Thangavel 2007; Udoeyo et al. 2010; Kanchan and Jawaid 2013; Kawade, Rathi, and Girge 2013; Apiwaranuwat et al. 2013; Muanglong et al. 2013; Shafana and Venkatasubramani 2014; Yashwanth 2014). However, all these studies were carried out mainly with regard to the material physical properties and very few studies have been carried out on the effects of the use of these materials on structural behaviour of the elements made from these materials. Olutage, Adeniran, and Oyegbinle (2013) investigated the ultimate capacity of laterised concrete with five classes of specimens incorporating 0, 10, 20, 30 and 40% laterite replacement of sand and reinforcement of 2, 3, 4 and five numbers of Y10 for each specimen class. Their results showed that laterised concrete gave a satisfactory performance compared with normal concrete when the replacement does not exceed 25%. Salau and Balogun (1990) investigated the shear resistance of laterised concrete beams without shear reinforcement and observed that the mode of failure does not depend on the percentage laterite content but mainly on the shear span of the beam. They further identified that the ultimate cracking load decreases with increase in percentage laterite content and the ultimate shearing stress of laterised concrete compare favourably with that specified in the code of practice CP110 1987. A different study by Salau and Sharu (2004) investigated the behaviour of bamboo strips as reinforcement on 75 structural columns made with laterised concrete. They found that the reinforced laterised columns could sustain increased deformation and strain with superior post-yield and post-cracking behaviour. In order to utilise a material in structural elements, it is important to understand the mechanical properties of that material before using it them in construction of structures.

Very few researches have been carried out on the combine utilisation of sugarcane bagasse ash and laterite soil in concrete. Shuaibu, Mutuku, and Nyomboi (2014) investigated the combine effect of sugarcane

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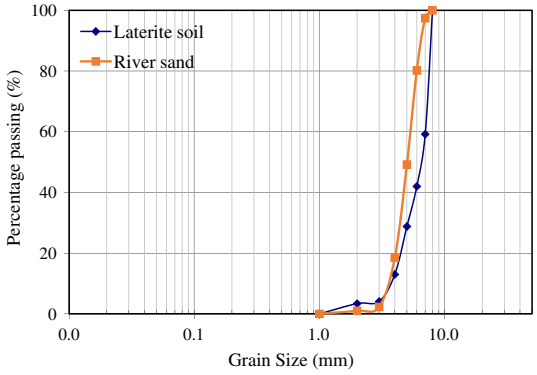

Figure 1. Particle size distribution of the aggregates sand laterite.

Table 1. Mix proportions.

Mix	Cement (kg/m^3)	Sugarcane bagasse ash (kg/m^3)	Sand (kg/m^3)	Laterite (kg/m^3)	Coarse aggregate (kg/m^3)	Water (kg/m^3)
SB-LAB-00–00ys/ns1.2–00	343	0	686	0	1371	195
SB-LAB-05–05ys1.2–00	326	17	652	34	1371	216
SB-LAB-10–15ys1.2–00	309	34	583	103	1371	226
SB-LAB-10–15ys1.2–00	309	34	583	103	1371	215
SB-LAB-15–20ys1.2–01	291	51	549	137	1371	237
SB-LAB-15–20ys1.2–02	291	51	549	137	1371	237

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bagasse ash and laterite soil as replacement of cement and sand, respectively, on properties of concrete. Their studies showed that the introduction of sugarcane bagasse ash and laterite soil reduced the compressive strength of concrete. However, replacement of 20% of cement by sugarcane bagasse ash and 25% of sand by laterite soil produced a concrete that attained the targeted strength of 20 Mpa for 1:2:4 mix. Shuaibu et al. (2014) also investigated the behaviour of reinforced sugarcane bagasse ash-laterised concrete beams without shear reinforcement. Their findings showed that sugarcane bagasse ash-laterised concrete beams can be designed using the design provisions contained in BS 8110-1:1997. This study, however, complements the previous findings by investigating the effects of sugarcane bagasse ash and laterite and laterite soil on shear strength of beams cast with varying proportions of sugarcane bagasse ash and laterite soil. Concrete mix ratio of 1:2:4 with a slump value of 30 ± 2 mm was used throughout the experiments. The performance of sugarcane bagasse ash-laterised concrete beams in shear was compared with that of normal concrete control beams made with cement, sand and coarse aggregates. Shear failure is often seen as sudden and catastrophic in nature and occurs when the principal tensile stress limit in the shear region is exceeded. Although, there are design procedures for normal reinforced concrete, there is need to investigate the behaviour of this composite material in order to build confidence in its utilisation in structural applications.

Materials and methods

Materials

The materials used in this study were as follows:

(a) Coarse aggregate (crushed stone) with a specific gravity value of 2.8, water absorption of 3.4% and moisture content 1.9%, sourced locally in Kenya.
(b) River sand sourced from Masinga dam in Kenya with a specific gravity of 2.6, water absorption of 0.45% and a moisture content of 0.25% was used.
(c) Laterites obtained from Juja area in Thika sub-county, Kenya, with specific gravity of 2.5 according to ASTM 128-12 2003 and a moisture content of 6.3%.

The particle size distributions of the laterite and sand are shown in Figure 1.

Ordinary Portland cement from Bamburi Cement Limited, Kenya, known as PowerPLUS 42.5 was used, which conforms to the specification of BS 12 1996 and is formulated from Portland Cement Clinker and interground with other constituents, in accordance to the requirements of EN 197-1 2000 and is used in medium to large construction constructions projects. Sugarcane bagasse ash was sourced from Mumias Sugar Company

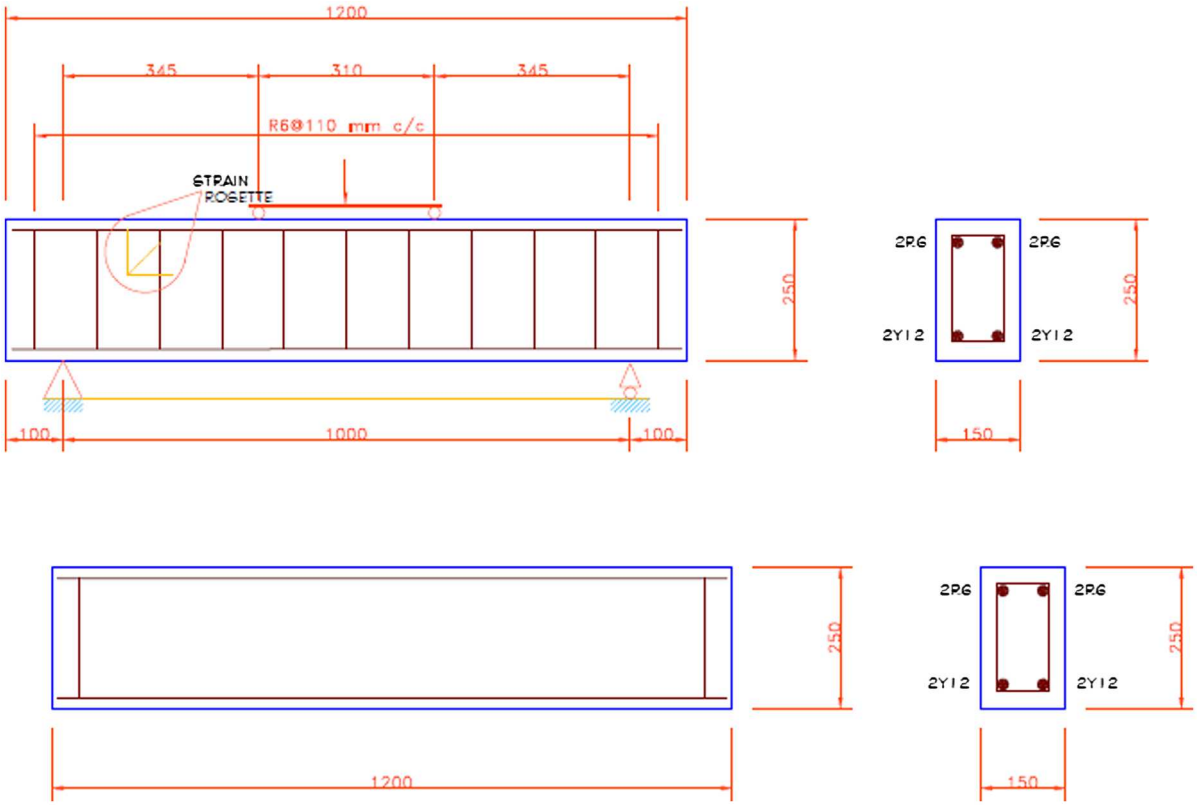

Figure 2. Beam specimen details and set-up.

in WesternKenya. The ash passing the sieve of aperture size of 75 μm was utilised and had a density of 2.3 g/cc. Potable Portable water conforming to the requirements of BS 3148 1980 was used. Table 1 shows the mix proportions used.

Concrete production, placement and curing

The concrete produced in this study was based on a mix ratio of 1:2:4 and constant slump of 30 ± 2 mm in accordance with BS 1881:102 1983. Seven beam specimens were cast in steel iron moulds, de-moulded after 24 h and cured by wrapping them in membranes, by sprinkling water on the membranes daily for 28 days. Three concrete cubes from each beam concrete mix were also cast and cured by immersion in water for 28 days.

Beam specimen details

A total of seven beams were cast casted with one beam cast without shear reinforcement and the other six with shear reinforcement. The beam beams specimens were of 150 mm × 250 mm cross section and a total length of 1200 mm. All the beams were reinforced with 12-mm diameter high yield twisted bars (Y12) at the tension zone and 6-mm diameter round mild steel (R6) at the compression zone as a minimum reinforcement to keep the reinforcement cage in place with a clear cover of 20 mm. Vertical 20-mm vertical stirrups of 6-mm diameter mild steel (R6) were used for the six beams with shear reinforcement at a spacing of 110 mm centre to centre. The specimen details and set-up are shown in Figure 2.

Electrical resistance strain gauges were attached to the centre of the longitudinal reinforcement to measure the induced flexural strains and on the shear reinforcement 240 mm from one edge of the beam but centred both vertically and horizontally on the stirrups before casting of the beams to measure the induced shear strain on the stirrups. The different replacement and specimen designations used in this study were as follows:

(a) SB-LAB-00-00A-00
(b) SB-LAB-00-00-00
(c) SB-LAB-05-05-01
(d) SB-LAB-10-15-01 and 02 and
(e) SB-LAB-15-20-01 and 02

In the specimen specification ***SB-LAB-XX-YY-ZZ***, ***SB-LAB*** indicates sugarcane bagasse ash-laterised concrete beam made with XX replacement percentage of cement by sugarcane bagasse ash and ***YY*** percentage replacement of sand by laterite soil. ***ZZ*** indicates the serial number of the beam specimens. The overall length of each beam is 1.2 m and ***A*** indicates the absence of shear reinforcement .For control beams ***XX***, ***YY*** and ***ZZ*** were taken as ***00***.

Testing methodology

Three samples of 150-mm cube specimens for each beam, were tested in compression at 28 days in accordance with to BS 1881-116 1983 using a universal testing machine. The beams were white washed before testing

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Table 2. Summary of test results.

Specimen type	Average cube strength at 28 days (MPa)	Ultimate shear load (kN)	Ultimate shear stress (N/mm^2)	Flexural moment M_f (kN-m)	Ultimate shear moment M_u (kN-m)	Moment capacity
SB-LAB-00–00ns1.2–00	31.04	58.55	1.70	36.95	20.20	0.55
SB-LAB-00–00ys1.2–00	35.68	77.01	2.05	42.46	26.57	0.63
SB-LAB-05–05ys1.2–01	30.60	68.85	2.00	36.42	23.75	0.65
SB-LAB-10–15ys1.2–01	27.99	67.78	1.81	33.31	23.38	0.70
SB-LAB-10–15ys1.2–02						
SB-LAB-15–20ys1.2–01	24.88	67.23	1.80	29.61	23.19	0.78
SB-LAB-15–20ys1.2–02						

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was done in order for the cracks to be made visible. The set-up for the beam test is shown in Figure 2. The beams were simply supported over a span of 1000 mm and tested under third point loading using a hydraulic jack. The load from the hydraulic jack via a load cell was transferred to the beam loading points through the loading arrangement as shown in Figure 2. A displacement transducer was also placed at the centre of the beam mid-span as shown in the same figure to determine deflection at the centre of the beams under the applied load. Strain rosette was placed midway the shear span in both vertical and horizontal directions on the concrete surface. An effective depth of 230 mm and a shear span to effective depth ratio of 1.5 was maintained for all beams. Steel strain gauges were also placed at the centre of the longitudinal reinforcement and the shear link to measure the induced flexural strains and shear strains in the reinforcements under applied load.

All the strain gauges, the load cell and displacement transducer were connected to a TDS data logger for data acquisition. The beams were loaded gradually by means of the hydraulic jack up to failure and all the deflections and strains were measured at every load increment. The first crack loads were noted immediately after their propagation and marked as well as the other cracks formed as the loading increased.

Results and discussion

Test results and analysis

The test results for this study are summarised in Table 2. The values of the concrete compressive strength, the ultimate shear load, the ultimate shear strength and the moment capacities are contained in Table 2.

The flexural moment capacity M_f was calculated from the relation $M_f = 0.156bd^2f_{cu}$, where f_{cu} is the compressive strength of the concrete as determined by the 28 days cube strength of the specimen, b is the breath of the beam and d is the effective depth of the beam specimen. The ultimate shear moment M_u was also calculated by the relation $M_u = P_{max}a_v/2$, where $P_{max}/2$ is the maximum shear load of the beam calculated as half of the maximum applied load, and a_v is the clear shear span of the beam.

The values in Table 2 show that the experimental ultimate shear moment decreases as the percentage replacement level of cement and sand increase. This can be due to the fact that the shear load also decreases with the replacement levels of the materials. The moment capacities are also shown in Table 2 for all the beams. The use of Equations (1) and (2) for the determination of moment capacity was also adopted by Salau and Balogun (1990).

Comparative analysis of shear load–deflection behaviour

Figure 3 shows the comparative response of the beams in terms of shear load–deflection curves. Table 3 shows the values of deflections at ultimate shear loads for an effective beam span of 1 m and varying percentage replacements of cement and sand. It can be observed that the deflections at ultimate shear loads continue to decrease as the percentage replacement level of both cement and sand increases. This may be due to the visco-elasticity of laterite indicating that the ductility of the beam increases as the percentage replacement of cement and sand increases. The service load was calculated in order to get the deflection at service load by dividing the ultimate applied load by a factor of 1.5 according to BS 6399-1 1996 for family dwellings not exceeding three storey building.

Shear stress–shear strain relationship

The experimental shear stress and shear strain for each beam sample including the control beam specimen were determined. The shear strain was calculated from the strain rosette arrangement using the relation in Equation (1)

$$\gamma_{xy} = \varepsilon_{45^\circ} - (\varepsilon_{90^\circ} + \varepsilon_{0^\circ}) \quad (1)$$

where γ_{xy} is the shear strain, ε_{45° are the strain values from the electrical strain gauges placed at 45°, ε_{90° are the

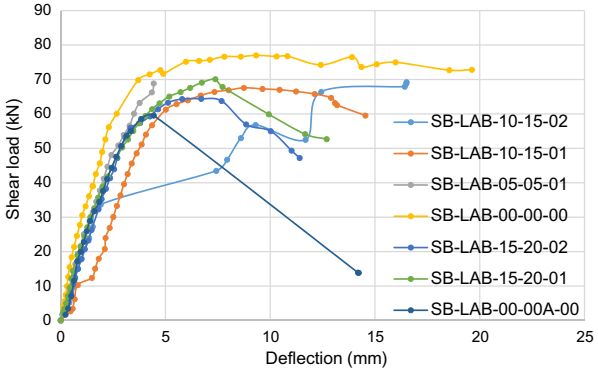

Figure 3. Comparative shear load–deflection curves.

Table 3. Deflection at various stages of beams.

Specimen type	Service load (kN)	Ultimate load (kN)	Deflection at service load (mm)	Deflection at ultimate load (mm)
SB-LAB-00–00-00	74.42	119.07	1.67	4.44
SB-LAB-00–00-00	96.26	154.02	1.70	9.33
SB-LAB-05–05-01	86.07	137.71	2.10	8.43
SB-LAB-10–15-01	85.57	136.91	2.30	8.07
SB-LAB-10–15-02				
SB-LAB-15–20-01	84.04	134.46	2.40	7.05

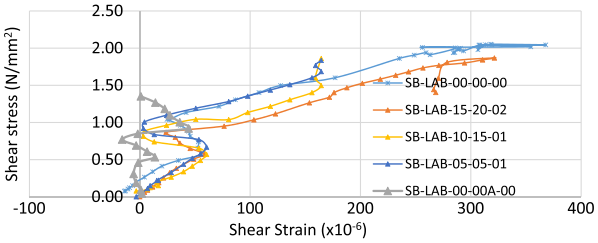

Figure 4. Comparative shear stress–shear strain behaviour of beams.

strain values from the electrical strain gauges placed at 90° and ε_{0° are the strain gauge values from the electrical strain gauge placed at 0°.

The shear stress was calculated from the relation

$$v = \frac{P}{2bh} \tag{2}$$

where P is the instantaneous applied where P the instantaneous is applied load, b is the breath of the beam and h is the overall height of the beam. The relations in Equations (1) and (2) were also adopted by Nyomboi et al. (2010). The experimental shear stress against the shear strain is as shown in Figure 4.

The shear stress–shear strain relationships for all the beams follow a particular pattern as was observed in Figure 4 but the ultimate shear strength for the beams containing shear reinforcement decreased as the replacement level increased. It is noted further that there was strain negative relaxation between shear stress of 0.6 to 1 MPa. This could be attributed to the shear deformations at the onset as strut crushing takes place, upon which normal positive strain behaviour is sustained due to further resistance offered by the shear reinforcements. It was also observed that the 5% replacement of both cement and sand (SB-LAB-05-05) follows almost the same pattern as the control beam but does not achieve the same ultimate shear strain as the control beam. The beam without shear reinforcement attained a lower shear stress in comparison with the other beams with shear reinforcement. This is due to the fact that shear failure is a tensile failure, therefore shear reinforcements in the beams with shear reinforcement were able to resist higher tensile stresses whereby making it to fail at higher shear stress in comparison with beams without shear reinforcement. Table 2 shows the maximum shear strength of all the beams in comparison with the limits set by BS 8110-1 1997 not to exceed $0.8\sqrt{f_{cu}}$, that is 3.58 N/mm^2.

Crack pattern and propagation

The physical failure modes of the beams are shown in Figure 5.

Flexural cracks were formed in the early steps of loading in the pure bending region. As the loading increased, diagonal cracks appeared within the mid height of the beams within the clear shear span of the beams. While the diagonal cracks were developing across the length,

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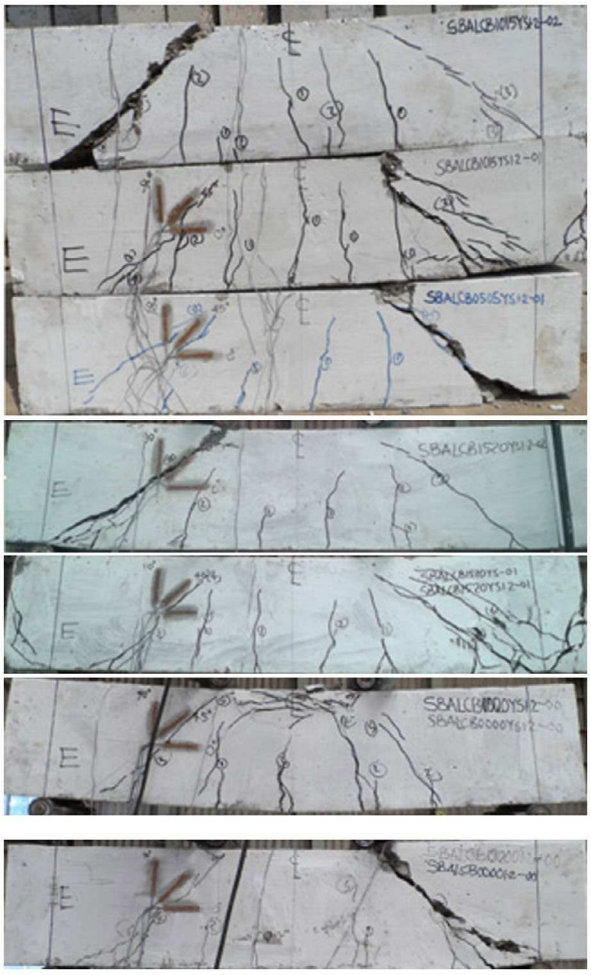

Figure 5. Physical failure patterns of beams.

their width was propagating within the shear span region. The failing mechanism identified was as follows:

(a) Diagonal splitting along the loading point was observed in some of the beams such as SB-LAB-05-05-01 and SB-LAB-10-15-02. This might be due to inadequate shear reinforcement that causes the diagonal crack to spread more rapidly resulting in collapse by splitting the beam into two parts in a diagonal tension.

(b) Shear–flexure was also observed in SB-LAB-00-00-00. This might be due to the fact that no replacement of cement and sand was made for the specimen. Therefore the bond between the aggregate and binder was uniform and the failure also was symmetrical shear-flexure.

(c) Due to the formation of several parallel diagonal cracks, strut crushing was also observed in SB-LAB-10-15-01 and SB-LAB-15-20-00. This might be due to the reduction of the tensile strength of the concrete arising from replacement of cement and sand thereby causing multiple diagonal tension cracks.

(d) Bearing failure was observed in SB-LAB-15-20-01 as a result of the bearing stresses exceeding the bearing capacity of the concrete. These stresses according to Subedi and Baglin (1999) are assumed to range from $0.7 - 0.85f_{cu}$.

(e) Shear-compression failure was observed for beams without shear reinforcement. The concrete in the compression zone was observed to be crush above the tip of the shear cracks. This might be due to inadequate reinforcement in the compression zone.

Comparison between theoretical and experimental results

The theoretical ultimate loads for third point loading arrangement calculated from the flexural moment by the relation in Equation (3) below as used by Altun et al. (2005) are compared to the experimental values in Table 4.

$$P_u = 3M_f/L \tag{3}$$

The moment of resistance was taken as the flexural moment used in Equation (1) for all the beams. From the results summarised in Table 4, the variance between the experimental and the theoretical values of the ultimate load for the beam without shear was 7.42% but that

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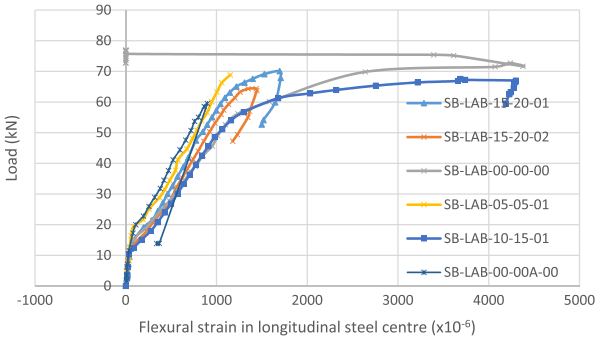

Figure 6. Shear load against flexural strain in longitudinal steel centre.

Table 4. Comparison between theoretical and experimental ultimate loads.

Specimen type	Experimental ultimate load $P_{u(exp)}$ (kN)	Theoretical ultimate load $P_{u(Theo)}$(kN)	Percentage variance (%)
SB-LAB-00–00-00	119.07	110.84	7.42
SB-LAB-00–00-00	154.02	127.39	20.91
SB-LAB-05–05-01	137.71	109.26	26.04
SB-LAB-10–15-01	136.91	99.93	37.01
SB-LAB-10–15-02			
SB-LAB-15–20-01	134.46	88.82	51.38
SB-LAB-15–20-02			

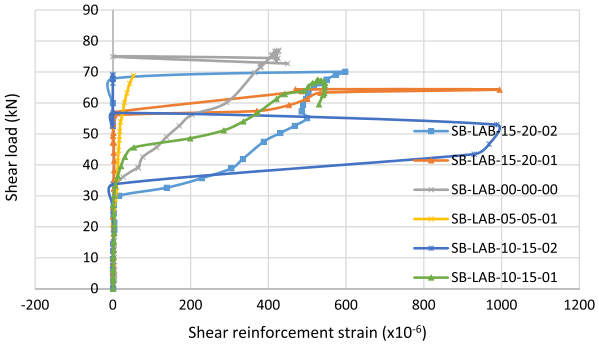

Figure 7. Shear load against shear reinforcement strain.

of the beams with shear reinforcement increased up to 51.38%. This indicates that application of the theoretical formulae in design would give safe but rather uneconomical predictions of the structural capacity. However, applying the theoretical formulas give a more economical prediction for the beam without shear reinforcement.

Strain induced within the beams

The values for the flexural strains induced at the centre of the longitudinal reinforcement and the shear strain at the centre of the shear reinforcement placed at a point 240 mm from the extreme edge of the beam and 135 mm from the beam bottom are plotted against the applied shear load as shown in Figures 6 and 7, respectively.

From the experimental shear load–strain behaviour, shown in Figures 6 and 7, the zigzag pattern displayed by SB-LAB-00-00 in Figure 6 and SB-LAB-00-00, SBLAB-10-15, SB-LAB-15-20 in Figure 7 indicates that the beams exhibited relaxation when ultimate shear load was attained. In Figure 6, the strain response of all the beams initially followed a nearly linear relationship as the load continues to increase between 10 and 60 kN; the SB-LAB-00-00-00 continued to sustain more load until the concrete snapped and the steel undergoes relaxation at ultimate applied shear load as shown by the curve pattern. At shear load between 30 and 40 kN in Figure 7, the beams response in terms of the strain induced in the shear reinforcement began to differ as can be seen from the curve. It can also be seen that the beam without shear reinforcement SB-LAB-00-00A-00 sustained the lowest load before relaxation.

Conclusions

The comparative shear resistance of sugarcane bagasse ash-laterised concrete has been experimentally studied with seven reinforced beams cast with various replacement levels of cement and sand by sugarcane bagasse ash and laterite soil, respectively, and tested under third point loading up to failure loads for shear behaviour. The following conclusions may be drawn from this study:

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(i) The 28-days compressive strength shows that the replacement levels of cement and sand by sugarcane bagasse ash and laterite, respectively, reduced the concrete cube compressive strength. However, the targeted strength for 1:2:4 mix proportion of 20 MPa was exceeded by various levels of replacement used in this study.

(ii) From the beam tests, it was observed that the experimental ultimate shear load of the beams reduced as the replacement level increased. However, all the ultimate shear stresses were below the limit set by BS 8110-1:1997 for shear stress.

(iii) The deflections for all the beams at service loads were found to be less than those specified by BS 8110-1:1997 (not exceeding span/250 ratio).

(iv) Although, the control beam with shear reinforcement SB-LAB-00-00-00 gives a higher shear stress as compared to other beams with cement and sand been replaced by sugarcane bagasse ash and laterite soil, respectively; all the beams up to 15% replacement of cement by sugarcane bagasse ash and 20% replacement of sand by laterite soil gave higher shear strength than the control without shear reinforcement SB-LAB-00-00A-00.

RECOMMENDATIONS

It is recommended that future research work be carried out on the effect of variation of span/effective depth ration on the shear strength of sugarcane bagasse ash-laterised concrete beams. Furthermore, it is also suggested that the durability sugarcane bagasse ash-laterised concrete is needed in order to complement the findings on the serviceability criteria.

Disclosure statement

No potential conflict of interest was reported by the authors.

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